DEVELOPMENT OF FLEXIBLE PAVEMENT DESIGN PARAMETERS FOR USE WITH THE 1993 AASHTO PAVEMENT DESIGN PROCEDURES

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16. Abstract

The American Association of State Highway and Transportation Officials published the Guide for Design of Pavement Structures (AASHTO Guide) in 1986, and updated in 1993. Design parameters for use in the flexible pavement design module of the computer program, DARWin^{TM}2.01, which is based on the 1993 AASHTO Guide were determined. Effective soil resilient modulus, layer coefficients, and drainage coefficients have been identified as three parameters essential to use the AASHTO Guide in Rhode Island.

Representative materials for the state of Rhode Island have been acquired and fundamental testing were done to determine their properties. All the materials showed good soil classification. A series of laboratory resilient modulus tests were performed on two granular subgrade soils at four temperatures and three moisture contents using the AASHTO T292-91 testing procedure. Prediction equations were developed to determine the resilient moduli under Rhode Island environmental and field conditions.

A procedure to estimate the cumulative 18-kip ESAL was developed utilizing the weigh-in-motion (WIM) data in Rhode Island.

Layer coefficients for bound and granular subbase materials were estimated using the method described in the AASHTO Guide. The coefficients for subbase materials ranged from 0.09 to 0.22 with an average of 0.15 and bound layer from 0.34 to 0.47 with an average of 0.39.

The drainage coefficient of the subbase materials were determined using the method recommended by AASHTO Guide. The drainage coefficient of the subbase materials ranged from 0.8 to 1.0 and the average is 0.9.

The estimated parameters were applied to the AASHTO Guide and DARWin^{TM}2.01 software which resulted thicknesses for each layer of the typical flexible pavement structures in Rhode Island.

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Preface

This is a research report of the project, entitled "Development of Flexible Pavement Design Parameters for Use with the 1993 AASHTO Pavement Design Procedures." The primary objective was to develop specific design parameters for flexible pavement structures in Rhode Island for use with DARWin™ 2.01, which is based on the 1993 AASHTO Guide and is currently being used by Rhode Island Department of Transportation (RIDOT).

The research project was conducted by the Department of Civil and Environmental Engineering at the University of Rhode Island (URI) under the contract No. SPR-223-2227 (RIDOT-RTD-95-3) to the Rhode Island Department of Transportation (RIDOT). Funds were provided by RIDOT through the Federal Highway Administration (FHWA), U.S Department of Transportation.

The work presented herein was accomplished by a team including Prof. K. Wayne Lee, Dr. Alan Marcus, Mr. Sekhar Vajjhala, Mrs. Karissa Mooney, Mr. Edgar Kraus and Mr. Kyungwon Park. Appreciation is due Messrs Brian Gray, Kevin Broccolo, Todd Brayton, Nikone Soupharath, Amphone Soupharath, and Yong Cao who assisted with laboratory work. Grateful recognition is forwarded to Mrs. Virginia Mulholland, Mrs. Gail Paolino and Mrs. Kimberley Boisclair for manuscript preparation and administrative assistance.

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CHAPTER 1. INTRODUCTION

For over a century, paved roadways have been constructed using asphalt concrete mixtures in Rhode Island as well as across the United States. A significant number of studies have been conducted to develop a comfortable, safe and economical pavement system. However, a major problem still exists involving premature distresses and pavement failures. In recent years this difficult problem has been further aggravated by substantial increase in loads transmitted by modern heavy trucks (Lee et al. 1990).

Recognizing the above problem, an AASHTO Joint Task Force on Pavements was formed to rewrite the Interim Guide for the Design of Pavement Structures (Interim Guide) published in 1972 incorporating new developments and specifically addressing pavement rehabilitation. Because many states including Rhode Island were found to be using at least portions of the Interim Guide, the 1986 Guide retained the basic algorithms developed from the AASHO Road Test. Because the Road Test was very limited in scope, i.e., a few materials, one subgrade, non-mixed traffic, one environment, etc., the framework was further expanded in the 1986 Guide such that designers could consider other conditions. The Task Force also recognized that a considerable body of information exists to design pavements utilizing so-called mechanistic models. It is further believed that considerable improvements will occur as these models will be calibrated to in-service performance, and are incorporated into everyday design usage. AASHTO also revised the 1986 Guide, and published the AASHTO Guide for Design of Pavement Structures (AASHTO Guide) in 1993.

Because AASHTO wanted to provide state of the art approaches without lengthy research, the AASHTO Guide includes values and concepts that have limited support in research or experience. Therefore, each user should consider this to be a reference document and carefully evaluate the need of each concept and what initial values to use. Therefore, the present study was carried out to develop pavement design parameters for the Rhode Island and environmental conditions.

Because Rhode Island Department of Transportation (RIDOT) constructs mostly asphalt pavements for new and rehabilitation projects, it had been initially identified that following three parameters are essential to use the AASHTO Guide for design of flexible pavement structures in Rhode Island:

1. Seasonal variation of soil resilient modulus
2. Layer coefficients of pavement materials
3. Drainage coefficients of pavement materials

A research team at the University of Rhode Island (URI) provided a preliminary procedure to determine the effective roadbed soil resilient modulus (Kovacs et al. 1991), and also layer coefficients (Lee et al. 1994a) to the RIDOT.

In 1990, RIDOT attempted revising its 1984 Design Procedure for Flexible Layered Pavements, and published "Design Procedure for Pavements." Although the 1990 RIDOT design procedure includes some features of 1986 AASHTO Guide including use of software, DARWin™1.0, it was neither comprehensive nor implementable. Actually, RIDOT abandoned the 1990 procedure, and recommended the use of the AASHTO Guide and/or DARWin™2.01 for design of any asphalt pavement structures in Rhode Island.

Since the URI research team has successfully developed a framework to determine effective soil resilient moduli and layer coefficients for Rhode Island soils and materials according to the AASHTO Guide, it appears to be ready to develop more accurate parameters for use with the AASHTO Guide and/or DARWin™2.01. However, the final phrase of this endeavor needed to include following tasks:

(1) Utilization of the AASHTO Designation T292-91 to determine resilient moduli of subgrade soils and untreated subbase materials

(2) Determination of layer coefficients for the cold recycled base layer materials

(3) Determination of drainage coefficients, m_i

(4) Traffic analysis procedure to estimate 18-kip ESALs for functionally classified roads: typically, local, collector, arterial and freeway

(5) Incorporation of the effects of the environment on the pavement performance analysis.

Therefore, the present research project examined, but was not limited to, the use of AASHTO Designation T292-91 method to determine the effective roadbed soil resilient modulus; and determined drainage coefficients, in addition to layer coefficients, for Rhode Island materials and conditions. When pavements will be designed using the AASHTO Guide along with the proper design parameters provided by URI research team, it is our hope that RIDOT can provide pavements in a serviceable condition over a given period of time at the least cost.

CHAPTER 2 CURRENT STATUS OF KNOWLEDGE

2.1 Soil Resilient Modulus

Subgrade soils have a major impact on the design, construction, structural response, and performance of a pavement. All pavement structural design procedures require a subgrade soil input (i.e., CBR, soil support number, resilient modulus, k and R value etc.). Unstable subgrade presents problems relative to placing and compacting subbase and base materials and providing adequate support for subsequent paving operations. Without an adequate "working platform" critical pavement construction details may not be accomplished within acceptable tolerances. Frequently, such construction deficiencies are undetected because they are hidden in the finished pavement. A large percentage of the surface deflection of a pavement is accumulated in the subgrade. Adequate subgrade characterization requires consideration of the fluctuation of subgrade soil properties as a function of space (various location with depth in the subgrade soil properties and longitudinal location along the project) and time (seasons of the year and yearly climatic variation).

The major recent emphasis in subgrade soils and granular material evaluation has been repeated load testing. Resilient modulus and permanent deformation can be quantified based on appropriate repeated load testing data. However, the permanent strain accumulated per load cycle is very small compared to the total strain in a well-designed pavement system.

In the 1993 AASHTO Guide, resilient moduli are used to characterize subgrade soil and assign layer coefficient to granular subbase and base layers. However State Highway Agencies (SHAs) are experiencing considerable difficulty in establishing the appropriate resilient modulus inputs to design pavement structures.

2.1.1 Repeated Load Testing

Suggested procedures for repeated load tests have been proposed by several agencies. AASHTO had adopted two procedures T292-91 and T294-94. In these procedures triaxial test conditions (generally constant confining pressure) are used for granular materials. Cohesive soils can be tested in unconfined compression or under triaxial conditions. It may be noted that

AASHTO has recently adopted TP46. It will become official as soon as the next interim edition is published.

Pneumatic and electrohydraulic repeated load equipment has been successfully utilized. The equipment must be capable of producing load pulse frequency of approximately 15 to 30 times a minute. Specimen deformation over a portion, or in some cases the entire length, of the specimen is typically measured using LVDT. Figure 2.1 illustrates the response of a soil to a repeated load pulse.

The intent of many laboratory studies is to simulate the field conditions. For the resilient modulus, $M_{R,}$ the results should duplicate the dynamic loading that the pavement experiences and the confining pressure the soil undergoes, due to the effects of heavy vehicles. The resilient modulus is not only influenced by the dynamic load and the deviator stress, but also by the density, freeze-thaw cycles and method of compaction also affects the results. The resilient modulus is therefore the ratio of stress due to the dynamic load to the recoverable or resilient axial strain.

$$M_R = \sigma_d / \in_R \qquad (2-1)$$

where

σ_d = P/A = stress due to dynamic load, deviator stress,

P = applied axial load,

A = the cross sectional area,

$\in_R$ = Δ/L_g = recoverable or resilient axial strain,

Δ = axial deformation, and

L_g = gauge length.

2.1.2 Modulus of Granular Materials

Granular materials "stiffen" (or increase resilient modulus) as the stress state increases. Repeated load on granular soils and materials has demonstrated the highly significant effect on the resilient modulus results. The M_R for granular materials is calculated by

$$M_R = K_1 (\theta)^{K_2} \qquad (2-2)$$

where

K_1 & K_2 = experimentally derived factors, and

θ = bulk stress

$= \sigma_1 + \sigma_2 + \sigma_3$ ($= \sigma_1 + 2\sigma_3$ in the triaxial test)

A typical plot of M_R versus stress state is shown in Figure 2.2.

2.1.3 Parameters Affecting Resilient Modulus of Soils

Many factors that influence the resilient modulus have been identified, such as the number of stress applications, the stress intensity, the age at initial loading, the method of compaction, and the moisture content.

Thompson stated that, for a given compaction condition, the M_R is significantly correlated with the physical properties of soils, For granular soils, gradation, shape, angularity, surface texture (crushed/uncrushed), and moisture content are major factors; and for fine-grained soils, plasticity index, clay content, the specific gravity, and soil consolidation affect the resilient moduli. In addition, the degree of saturation is also a factor reflecting the combined effect of density and moisture content.

Yoder and Witczak (1975) reported that the resilient modulus of granular materials increased with decreasing saturation and increasing density and angularity of the particles. It was also observed that resilient modulus is highly correlated with sample preparation procedure, especially for granular materials.

The recent study in Rhode Island (Kovacs et al. 1991) about seasonal effects on the soil resilient modulus found that: (1) moisture content, dry density, temperature and bulk stress are four major variables which can be used to develop prediction equation; (2) the resilient modulus tends to decrease as the water content increases up to a certain bulk stress level; thereafter, it varies nonuniformly regardless of the water content; (3) For the constant bulk stresses and dry densities, the resilient modulus decreases as the water content increases at a constant temperature; and (4) the resilient modulus increases with decreasing temperature at a constant water content.

2.1.4 Role of Resilient Modulus in Pavement Design

The AASHTO Guide replaced the soil support value (S) derived from CBR value with its resilient modulus value. It also recommends that the resilient modulus test be the definitive test for characterization of roadbed soil for use in both flexible and rigid pavement design applications.

For subgrade soils, the AASHTO Guide requires the input of an effective resilient modulus, which accounts for the combined effect of all seasonal modulus values. The effective M_R (M_{Reff}) quantifies the relative damage that can be included in the overall design. AASHTO Guide provides a chart used for recording the subgrade soil resilient modulus value during a year. The M_{Reff} is a weighted value of the average relative damage. It has been recommended by AASHTO that the effective M_R value should be used only for the design of flexible pavements using serviceability criteria.

2.2 Layer Coefficients

A value for this coefficient is assigned to each layer material in the pavement structure in order to convert actual layer thickness into structural number (SN). The SN is an abstract number expressing the structural strength of a pavement required for given combinations of soil support (M_R), total traffic expressed in equivalent 18-kip single axle loads, terminal serviceability, and environment. This layer coefficient expresses the empirical relationship between SN and thickness and is a measure of relative ability of the material to function as a structural component of pavement. The following general equation for structural number reflects the relative impact of the layer coefficients (a_i) and thickness (D_i):

$$SN = \sum_{i=1}^{n} a_i D_i \tag{2-3}$$

Although the elastic (resilient) modulus has been adopted as the standard material quality measure, it is still necessary to identify (corresponding) layer coefficients because of their treatment in SN design approach. Though there is correlation available to determine the modulus from tests such as the R-value, the procedure recommended is direct measurement using AASHTO Method T274 for unbound subbase and base granular materials and ASTM D4123 for asphalt concrete and other stabilized materials. Research and field studies indicate that many factors influence the layer coefficients. Thus the agency's experiences must be included in estimating coefficient values. For example, the layer coefficient may vary with thickness, underlying support, position in the pavement structure, etc.

2.3 Drainage Coefficient

Drainage of water from pavements has always been an important consideration in road design; however, current methods of design have often resulted in base courses that do not drain well. This excess water combined with increasing traffic volumes and loads often leads to early pavement distress in the pavement structure. Water enters the pavement structure in many ways, such as through cracks, joints, or pavement infiltration or as groundwater from an interrupted aquifer, high water table, or localized spring. Effects of this water (when trapped within the pavement structure) on pavements include:

(1) reduced strength of unbound granular materials,

(2) reduced strength of roadbed soils,

(3) pumping of concrete pavement with subsequent faulting, cracking, and general shoulder deterioration, and

(4) pumping of fines in aggregate base under flexible pavements with resulting loss of support.

Prior editions of the AASHTO Guide for Design of Pavement Structures have not treated the effects drainage on pavement performance. In the 1993 AASHTO Guide, drainage effects are directly considered in terms of the effect of moisture on roadbed soil and the effect of moisture on subgrade strength and on base erodability (for concrete pavements). Though consideration for stripping of asphalt concrete is not directly considered, the effects of swelling soils and frost heave are considered.

Figure 2.1 Repeated Load Testing Concepts

Figure 2.2 Resilient Modulus - θ Relation for a Sandy Gravel

CHAPTER 3 DETERMINATION OF EFFECTIVE RESILIENT MODULUS FOR SUBGRADE SOILS

The AASHTO Guide requires using the effective soil resilient modulus (M_R) as an input to design the flexible pavement structure. To determine the effective M_R, the seasonal variation on characteristics of subgrade soils is considered as a major parameter. A series of experiments was conducted in the present study to incorporate seasonal variation on soil M_R; and consequently to develop prediction equation (s) for estimation of the effective M_R.

3.1 Sample Collection

Subgrade soils from eight road sites were selected to represent a wide range of soil distribution in Rhode Island as shown in Figure 3.1. A description of each site is listed in Table 3.1. Materials from sites 1 to 3 were obtained from old study sites with existing pavements, used in phase I study. Materials from 4 to 8 were obtained from new construction projects, used in phase II study.

Because the original soil samples from sites 1 (Rt. 2) and 2 (Rt. 146 N) were depleted, substitutes were used in the present study to develop new prediction equations. For the site 1, two new soils along Rt. 2 were secured for substitute: (1) from the development site near the Barber Pond and (2) from the stockpile near the original test section. After examining gradation, AASHTO Soil Classification and moisture density relationship, the substitute soil from the development site (SG- 1S) was selected for further testing. Results of fundamental tests for the selection of substitute samples are summarized in Appendix A1. For the substitute of Rt. 146 (N) samples, i,e., SG-2, the Rhode Island Department of Transportation (RIDOT) allowed to use the soil samples from Rt. 146 site, i.e., SG-8. The sample from site 3, Upper College Road (UCR), was taken from the URI library construction site. For the five phase II soils same samples collected from the construction sites were used in the present study.

3.2 Fundamental Properties of Subgrade Soils

3.2.1 Soil Classification

Sieve analysis and Atterberg limits tests were performed for each subgrade soil sample in accordance with procedures of AASHTO T27, T89 and T90. Then, all samples were classified using AASHTO soil classification, and Unified Soil Classification (USC) as shown in Table 3.2. From left to right, (1) Soil ID indicates that SG is used for subgrade, to identify the samples with the order of sites where the samples were received; (2) Site documents the location of the construction sites; (3) AASHTO Class presents the AASHTO soil classification ; (4) USC class shows the Unified Soil Classification; and (5) Passing No. 200 reports the soil's fine grain content. It was found that subgrade soils are A-1-b granular soils, and the percentage of passing No. 200 ranges from 7.2 (Jamestown) to 24.7 (Rt. 2 substitute).

3.2.2 Moisture Density Relationship

The moisture content and density relationship for a soil is a critical factor affecting the strength and deformation properties of any prepared soil. Therefore, careful laboratory testing to establish this relationship is critical to provide an accurate determination of the resilient modulus. A series of tests to determine the Optimum Moisture Content (OMC) and maximum dry density (γ_d) of each subgrade soil sample was conducted based on the AASHTO T180 (using a 10-lb rammer and an 18-in drop). The results are summarized in Table 3.3.

3.2.3 The California Bearing Ratio Test

The California Bearing Ratio Test (CBR) test (AASHTO T193-81) measures the resistance of a soil molded at its optimum moisture content and then soaked prior to being tested as a step in evaluating the ability to adequately sustain traffic loads. CBR test was performed in this study to investigate the correlation, if any, between CBR and resilient modulus. Table 3.3 summarizes the CBR values.

3.3 Determination of Resilient Moduli for Subgrade Soils

AASHTO Guide recommended using the procedure of AASHTO T274-82 to determine the soil resilient modulus. In this procedure, a cylindrical specimen of soil is confined in a triaxial cell which allows varying confining pressures to be applied on the specimen to simulate the field conditions. A suitable loading system is used to apply a repeated load pulse of a fixed magnitude and fixed time duration. The deformation of the specimen is measured through linear variable displacement transducers (LVDTs), and is recorded for analysis. However, in the previous URI study, it was observed that most specimens were failed before collecting the data, when the AASHTO T274-82 method was used. Therefore, there was a definite need to develop a new procedure.

Based on the AASHTO T274-82 method, as well as procedures of ASTM, Oregon State University and other agencies, an improved resilient modulus testing method, i.e., URI method was developed (Kovacs, Lee & Jin 1991). The new loading sequence was selected through better simulation of field condition, and was implemented using the H & V testing machine for typical subgrade soils in Rhode Island.

In addition to the URI improved method, SHRP Protocol P46 (AASHTO T294-94) "Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils" method was also used in the previous study (Lee, Marcus & Mao 1994). It may be noted that there was no significant difference in test results between the URI improved and SHRP P46 methods. However, the AASHTO T292-91 procedure was used in the present study, upon RIDOT's suggestion.

3.3.1 Preparation of Test Specimens

Due to the depletion of original Rt. 2 and Rt. 146 (N) soils, substitute soils were used in the present study as mentioned in Section 3.1. The soil was mixed thoroughly with water to its OMC. This mixture is placed in a plastic bag for sealing and is stored at an atmosphere of at least 75 percent relative humidity for 24 hours. Complete sealing is ensured by storing the soil mixture in 2 or more bags.

After the storage period, the soil is placed in a split mold purchased from H & V Inc. in 5 equal layers with 25 blows per layer using a 5.5 lb rammer and 12-inch drop to obtain a specimen of 4 -in diameter and 8 -in height. With this compaction procedure a 90 % or higher of maximum dry density (determined by AASHTO T180) was usually achieved.

3.3.2 Resilient Modulus Testing Procedure

The H & V test system used in the present study consists of (1) loading component (2) control cabinet, and (3) computer component. The vertical deformation caused by the repeated loads is measured by the LVDTs attached to the specimen inside the triaxial chamber. The signal control system allows the LVDTs and load cell to change through 12 gain settings depending on the material of a specimen; and also transfers the output signals from load cell and LVDTs to a PC where the signals are recorded and manipulated with a software program named Resilient Modulus (RM). Consequently, the resilient modulus of the specimen in terms of $M_R = K_1 \theta^{K2}$ is calculated and printed by the "RM" program.

The resilient modulus test measures the elastic modulus of a soil specimen under a dynamic loading condition. Therefore, this requires the removal of any deformation during the specimen conditioning phase. This process is accomplished by applying 1,000 repetitions of 12 psi deviator stress and 15 psi confining pressure as given in Table 3.4. The data collection phase starts immediately following the specimen conditioning phase using a testing sequence as shown in Table 3.4.

In addition, water is allowed to drain out from the bottom of a specimen during the testing and is collected for determining dry density and water content of the specimen after the testing.

3.3.3 M_R Test Results

The M_R tests were performed first at their OMCs at room temperature. The test results in the form of $M_R = K_1\theta^{K2}$ is summarized in Table 3.5. The coefficients of determination (R^2) were between 0.79 to 0.95 for AASHTO 292-91procedure. Additionally, soil types, actual moisture content and dry density after testing, and test temperatures are also indicated in this table. Previous results using the URI method have been included for possible comparison purposes.

3.3.4 Determination of Resilient Modulus for Subgrade Soils

To determine the soil resilient modulus, the bulk stresses were estimated at the average depth of significant stress (ADSS) (Lee et al. 1994a). The bulk stress for subgrade soil at ADSS, θ_{SG} can be calculated as

$$\theta_{SG} = \sigma_1 + \sigma_2 + \sigma_3$$

$$\theta_{SG} = \sigma_d + \sigma_s + 2K_0\, \sigma_S \qquad \text{(Eq. 3-1)}$$

where, σ_1, σ_2, and σ_3 are principal stresses

σ_d is deviator stress

σ_s is static stress

$K_0 = 1- \sin\phi$, for cohesionless soil and gravel and

ϕ is angle of internal friction.

The calculation of stresses was done using a multi layer elastic program, ELSYM 5 (Ahlborn 1972). The input data required are the thickness of the pavement, elastic modulus of each layer, Poisson's ratio of each layer and tire load and pressure on the pavement surface. The ADSS of subgrade soil was determined based on this ELSYM 5 stress calculation. The bulk stress of subgrade soil was computed at ADSS to represent the stress state in subgrade soils.

In order to determine more accurate M_R in subgrade soils, some lab determined values such as resilient moduli and densities of asphalt layers (asphalt surface, asphalt modified binder and asphalt modified base) were used as the initial input data for calculating the stresses. To obtain the initial input resilient moduli of subbase and subgrade, the MICH-PAVE computer program that enabled the computation of the equivalent resilient moduli of subbase and subgrade based on lab determined M_R equations was utilized in this project (Harichandran, et al. 1990). The equivalent resilient moduli then were input into ELSYM5 to determine the deviator stresses at the ADSS of subgrade. The static stresses were computed based on the material density of each layer.

The input data for ELSYM5 was summarized in Table 3.6. A 9 kip wheel load with 100 psi tire pressure was used. For phase II sites (4 to 8), the lab determined resilient modulus data was used to determine resilient moduli for subbase and subgrade. Poisson's ratios of 0.35 and 0.40 were assumed for asphalt and granular layers, respectively. Densities of subbase and subgrade were assumed as 95% of their maximum dry densities. The angle of internal friction of 30^{o} was assumed for subgrade soil and 40^{o} for subbase granular material to compute the confining stresses.

The results including ADSSs, stresses and resilient moduli are presented in Table 3.7 for each site. The resilient moduli of subgrade soils vary from 9.3 to 14.5 ksi (mean = 9.8 ksi).

3.4 Determination of Effective Soil Resilient Modulus

3.4.1 Seasonal Variation of Subgrade Soil Resilient Modulus

The effective M_R is required to determine structural number (SN) in the AASHTO Guide. Therefore, it was imperative to study the seasonal variation of resilient modulus in

Rhode Island. For this purpose, aforementioned soils from Rt. 2 and Rt. 146 sites were used in the present study, i.e., SG-1S and SG-8, respectively.

In order to study the seasonal variation of soil resilient moduli, the soil moisture-temperature cells (Soil Test MC-300B) were installed underneath the pavement of Rt. 2 and Rt. 146 site in April 1990 (Kovacs, Lee and Jin 1991). Moisture content and temperature data have been collected every month from 1990 to 1993, and the results are summarized in Appendix A2.

3.4.2 Experimental Design For Laboratory to Study Seasonal Variation

The seasonal variations of soil resilient moduli have been studied in the laboratory in order to establish a relationship between resilient modulus and environmental effects, as well as soil characteristics and also to determine the effective M_R value of subgrade soil.

Although the water contents obtained from the field ranged from 5.0 to 12.7 % on Rt. 2 and from 5.3 % to 9.7% on Rt. 146, it was found impossible to compact the specimen at some of these high moisture contents. Therefore, it was decided to develop regression equations to determine M_R at any moisture contents. Table 3.8 and Table 3.9 represent the experiment design for Rt. 2 and Rt. 146 soils, respectively.

Considering the effects of soil dry density on its resilient modulus two different compaction efforts were used to prepare a specimen at room temperature 25- and 35-blow per layer. These compaction efforts with 5.5 lb rammer and 12 in. drop are expected to produce a specimen that is at least 90% relative compaction with respect to its maximum dry density determined by AASHTO T-180. An environmental chamber purchased from Standard Environmental System Inc. was used to conduct the soil resilient modulus test at different temperatures.

3.4.3 Experimental Observations and Results

Although Rt. 2 soil could be tested at moisture content of OMC +2%, Rt. 146 soil could not be tested at the same water content. The Rt. 146 soil appeared to swell during compaction and the specimen broke even before the actual loading could be applied. Therefore, for Rt. 146 soil, the moisture content of OMC +1% was used in the experiment.

The resilient moduli varied depending upon the deviator stress, but regardless of the confining pressure. It was found that at higher moisture content there was low coefficient of determination (R^2) value which means poor correlation between the data and the regression model $M_R = K_1\ \theta^{K2}$. Hence we have R^2 values ranging from 0.57 to 0.97 for Rt. 2 and 0.61 to 0.98 for Rt. 146. Tables 3.10 and 3.11 summarize the experimental test results at different temperatures and different blows for Rt. 2 and Rt. 146 soils, respectively.

3.4.4 Statistical Analysis for Experimental Data

In order to predict the resilient modulus under various environmental conditions, a multi linear regression analysis was conducted with three different groups of laboratory data using a Statistical Analysis System (SAS) computer program.

Based on the experimental design, only bulk stress (θ), temperature (T), moisture content (w/c) and dry density (γ_d) were considered as major factors affecting the soil resilient modulus under different environmental conditions. Therefore, the resilient modulus was taken as the dependent variable, while the bulk stress, temperature, moisture content and dry density were taken as independent parameters. An interaction term, T*(w/c) was also introduced into the regression model based on the preliminary analysis of the raw test data (Hutchinson 1993). Consequently, the regression model used in this study had the following form:

$$\log M_R = a_0 + a_1 \log\theta + a_2 (w/c) + a_3 \gamma_d + a_4 T + a_5 T^* (w/c) \qquad \text{(Eq. 3.2)}$$

where a_0 to a_5 are regression coefficients.

The average monthly temperature and FWD backcalculated moduli indicated that normally the subgrade soils, at the depth of ADSS, on both Rt. 2 and Rt. 146 sites do not freeze during the Winter (Lee et al. 1994a). Therefore, the present study developed prediction equations (1) normal condition, (2) near frozen condition, and (3) normal and near frozen condition. The regression equations generated from SAS program are listed in Table 3.12 for Rt. 2 and Rt. 146 soils.

3.4.5 Effective Soil Resilient Modulus

In order to incorporate environmental effects for pavement design, an effective resilient modulus equivalent to the combined effects of all the seasonal moduli has been used, particularly in the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO Guide). Following is a brief description of effective M_R in the AASHTO Guide.

The basic design equations used for flexible pavements in the AASHTO Guide is as follows:

$$\log_{10}(w_{18}) = Z_R x S_0 + 9.36x\log_{10}(SN+1) - 0.20 + \frac{\dfrac{\log_{10}(\Delta FPSI)}{(4.2-1.5)}}{0.40 + \dfrac{1094}{(SN+1)^{5.19}}} + 2.32x\log_{10}(M_R) - 8.07$$

(Eq. 3.3)

where,

w_{18} = predicted number of 18-kip equivalent single axle load applications (ESAL)

Z_R = standard normal deviate

S_0 = combined standard error of the traffic prediction and performance prediction

ΔPSI = difference between the initial design serviceability index, p_0 and the terminal serviceability index, p_t, and

M_R = resilient modulus (psi)

SN = structural number

This equation may be separated into two parts: T_i, which represents the overall M_R effect on the predicted performance; and Q, which is independent of M_R:

$$T_i = 2.321\log_{10}[M_R] - 8.07$$

(Eq.

3.4)

$$Q = 9.36\log_{10}(SN+1) - 0.2 + \frac{\log_{10}\left[\frac{P_0 - P_t}{4.2 - 1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}}$$

(Eq. 3-5)

The subscript i in Eq.3.4 represent the i^th^ time increment during a year. Consequently, M_{Ri} represents the resilient modulus during the i^{th} time increment. Seasons are defined according to the number of time increments they occupy during a year.

Substituting T_i and Q into the original performance equation Eq.3-3 results in the following relationship:

$$W_{18_i} = 10^{Q} * 10^{T_i}$$

(Eq. 3-6)

Where 10^{Ti} represents the relative damage effect on the resilient modulus for a predicted performance. Therefore, the relative damage value, u_f, is defined as:

$$u_f = 10^{-T_i} = 10^{-[2.32\log_{10}M_{Ri} - 8.07]} = 1.18 * 10^{8} * M_{Ri}^{-2.32}$$

(Eq. 3-7)

Assuming that Miner's linear damage hypothesis is valid and that the rate of traffic application during each season of a given year is constant, the total damage, D, to a pavement during its life is given by:

$$D = \sum_{i=1}^{n} \frac{\left[\frac{W_{18_{TOT}}}{n}\right]}{W_{18_i}} = 1.0$$

(Eq. 3-8)

where, W_{18TOT} = total 18-kip ESAL of predicted traffic during the analysis period;

n = number of equal time periods into which the year is subdivided in order to identify the individual seasons.

$$W_{18_{TOT}} = \frac{10^Q}{\frac{1}{n}\sum_{i=1}^{n} u_{f_i}}$$

(Eq. 3-9)

substituting u_{fi} into the above equation yields the result:

The denominator in the equation indicates that the overall effects of the seasonal variation of subgrade resilient modulus can be described in terms of an average relative damage:

$$\overline{u_f} = \frac{\sum u_{fi}}{n} \qquad \text{(Eq. 3-10)}$$

The effective subgrade soil resilient modulus, then, is considered basically a unique resilient modulus that produces the same overall damage as the combined effects of the modulus during each season.

Based on the bulk stress, the dry density, and average monthly temperature and moisture content of subgrade soil, the resilient modulus at each month was estimated using the prediction equations as listed in Table 3.12. Then the relative damage factor was computed from Eq.3-7 for each month and the average relative damage factor was determined. The effective resilient modulus was estimated in accordance with the AASHTO Guide procedure.

The monthly resilient moduli of Rt.2 and Rt.146 subgrade soils were determined directly using the field temperature and moisture content data. The average monthly temperature and moisture content measured at ADSS were used. The bulk stresses used from Table 3.7 were 10.02 psi for Rt.2S and 10.79 psi for Rt.146. The dry density was assumed as constant during a year; and the maximum dry density determined in the lab was used.

The monthly resilient moduli for Rt.2S and Rt.146 sites were computed not only using the individual equation but also combined equation. Effective resilient moduli were determined using equations (3) and (6) in Table 3.12; and the moduli are 10,830 psi for Rt.2S and 6,440 psi for Rt.146. In addition, sample calculations using the prediction equation (9) in Table 3.12 are given in Tables 3.13 and 3.14 for Rt.2 and Rt. 146 subgrade soil, respectively. Effective resilient moduli determined from equation (9) were9,304 psi for Rt.2S and 8,782 psi for Rt.146.

For other sites, two assumptions were made for calculating the effective resilient modulus, since the field temperature and moisture content data is not available on these sites:

(1) Based on the location, for the sites that are in the southern part of the State,

the temperature-moisture data from Rt.2 was used to represent the field environmental condition for these sites. These sites include Upper College Road, Roger Williams Way, and Jamestown (Rt.138 east). Consequently, the sites in the northern part of State were used the data from Rt.146 and these sites are Rt.107, Charles Street, and Rt.146 south.

(2) The prediction equation based on the combined data at normal condition was used, i.e., equation (9) in Table 3.12.

Table 3.15 is a summary of effective resilient moduli for each site. It was observed that, by using the prediction equation, the variation of effective resilient moduli among all sites was mainly depending upon the soil dry density and bulk stress, since the variations of temperature and moisture content were similar among sites. On the other hand, dry density is the only parameter remaining in the prediction equation, which relates to the physical properties of subgrade soil. Other parameters mainly associate with traffic and environmental conditions where a pavement exists.

Table 3.1 Location of Each Study Site

Site	Study Phase & Identification No.	Location	Contractor
1	Ph I-1	**RT. 2,** North Kingston, near RT. 102.	D’Ambra Construction Co.
2	Ph I-2	**RT. 146 North,** North Smithfield, near the border of MA.	Tilcon Gammino, Inc.
3	Ph I-3	**Upper College Road,** Kingston, URI	J.H. Lynch & Sons, Inc.
4	Ph II-1	**Roger Williams Way,** North Kingstown, near Quonset Point.	Tilcon Gammino, Inc.
5	Ph II-2	**RT. 107,** Burrillville, Main Street.	J.H. Lynch & Sons, Inc.
6	Ph II-3	**RT. 138,** Jamestown, east end of new Jamestown bridge.	Cardi Construction Corp.
7	Ph II-4	**Charles Street,** Providence.	D'Ambra Construction Co.
8	Ph II-5	**RT. 146 South,** Lincoln.	Todesca (Forte) Corporation

Table 3.2 General Characteristics of Subgrade Soils

Soil ID	Site	AASHTO Class	USC Class	Passing No. 200
SG-1S	Ph I-1S Rt. 2	A-1-b	SW-SM	24.7%
SG-2		See SG-8		
SG-3	Ph I-3 Upper College Road, North	A-1-b	SM	13.7%
SG-4	Ph II-4 RWW	A-1-b	SP-SM	8.9%
SG-5	Ph II-5 Rt. 107	A-1-b	SP-SM	7.3%
SG-6	Ph II-6 Rt. 138, East	A-1-b	SW-SM	7.2%
SG-7	Ph II-7 Charles St.	A-1-b	SM	11.3%
SG-8	Ph II-8 Rt. 146, South	A-1-b	SC	20.8%

Table 3.3 Fundamental Test Results for Subgrade Soils

Soil ID	AASHTO Class	OMC %	Max. Dry Density pcf	CBR
SG-1S	A-1-b	6.7	131.1	22
SG-2		See SG-8		
SG-3	A-1-b	6.7	132.0	5
SG-4	A-1-b	8.7	121.5	9
SG-5	A-1-b	6.8	134.7	25
SG-6	A-1-b	8.5	127.5	9
SG-7	A-1-b	9.5	122.4	14
SG-8	A-1-b	5.9	133.1	11

Table 3.4 Loading Sequence of AASHTO T292-91 to Determine Resilient Moduli of Subgrade Soils

Phase	Sequence No.	Deviator Stress psi	Confining Pressure, psi	No. of Repetitions
Specimen Conditioning	1	12	15	1000
Testing	2	7	15	50
	3	10	15	50
	4	15	15	50
	5	5	10	50
	6	7	10	50
	7	10	10	50
	8	15	10	50
	9	3	5	50
	10	5	5	50
	11	7	5	50
	12	10	5	50
	13	3	2	50
	14	5	2	50

	15	7	2	50

Table 3.5 M_R Test Results for Subgrade Soils

Soil ID	Site	AASHTO T292-91 Method						URI Method					
		w/c%	γ_d pcf	T ^{0}F	K_1	K_2	R^2	w/c%	γ_d pcf	T ^{0}F	K_1	K_2	R^2
SG-1S	Rt. 2S	6.7	131.1	64	3141.9	0.57	0.95	No Test					
SG-2	Rt. 146N	See Rt. 146 South Soil Results						7.9	127.8	65	2864.1	0.69	0.93
SG-3	UCR	6.7	132.0	62	5196.1	0.51	0.80	6.4	132.1	75	2900.6	0.61	0.94
SG-4	RWW	8.7	121.5	68	2914.2	0.55	0.96	9.4	120.2	69	2271.4	0.69	0.97
SG-5	Rt. 107	6.8	134.7	61	8149.1	0.21	0.79	6.0	130.5	65	3960.6	0.59	0.95
SG-6	Rt. 138	8.5	127.5	69	No Material			8.7	124.3	66	2007.0	0.71	0.95
SG-7	Charles St.	9.5	122.4	69	No Material			10.3	120.6	64	3698.6	0.62	0.90
SG-8	Rt. 146S	5.9	133.1	65	2562.0	0.61	0.95	6.1	131.5	76	3422.6	0.61	0.96

Note: 1. $M_R = K_1\theta^{K2}$

2. R^2 = coefficient of determination

Table 3.6 Input Data for ELSYM5

Site	Layer	Thickness in.	Modulus psi	Poisson's*** Ratio	Density* pcf
Rt.2	Surface	1.5	325000*	0.35	145.3
	Mod. Binder	1.5	540000*	0.35	145.3
	Mod. Base 5.0		480000*	0.35	145.3
	Subbase	12.0	20400**	0.40	123.2
	Subgrade	-	14300**	0.40	126.7
Rt.146N	Surface	2.0	433000*	0.35	148.7
	Mod. Base 8.0		437000*	0.35	148.9
	Subbase	12.0	12000**	0.40	124.9
	Subgrade	-	12800**	0.40	124.9
UCR(N)	Surface	2.0	411000*	0.35	146.7
	Mod. Binder	2.0	372000*	0.35	145.5
	Mod. Base 3.0		497000*	0.35	147.1
	Subbase	12.0	22700**	0.40	126.1
	Subgrade	-	12100**	0.40	107.5
RWW	Surface	2.0	308000*	0.35	148.7
	Mod. Binder	2.0	371000*	0.35	146.0***
	Mod. Base 4.0		303000*	0.35	148.9
	Subbase	12.0	19000**	0.40	123.8
	Subgrade	-	10000**	0.40	115.1
Rt.107	Surface	2.0	318000*	0.35	146.7***
	Mod. Binder	2.0	410000*	0.35	145.5***
	Mod. Base 3.0		372000*	0.35	147.1***
	Subbase	12.0	16300**	0.40	122.2
	Subgrade	-	11800**	0.40	131.0
Jamestown	Surface	2.0	312000*	0.35	145.3***
	Mod. Binder	2.0	315000*	0.35	145.3***
	Mod. Base 5.0		381000*	0.35	145.3***
	Subbase	12.0	14400**	0.40	129.0
	Subgrade	-	12600**	0.40	119.7
Charles St.	Surface	7.0	291000*	0.35	145.0***
	Subbase	12.0	17900**	0.40	128.6
	Subgrade	-	11900**	0.40	116.5
Rt.146S	Surface	9.0	247000*	0.35	145.0***
	Subbase	12.0	15000**	0.40	132.6
	Subgrade	-	14700**	0.40	128.0

Note:
- -: Thickness of subgrade is assumed as semi-infinite.
- *: indicates that values were determined from laboratory prepared specimens.
- **: indicates that values were determined using the software MICHPAVE.
- ***: indicates that values were assumed.

Table 3.7 Resilient Modulus of Subgrade Soils at ADSS

Soil ID	Site	ADSS in.	σ_s psi	σ_d psi	$\sigma_3=\sigma_2$ psi	θ psi	M_R ksi
SG-1S	Rt. 2S	33.13	4.00	2.02	2.00	10.02	11.7
SG-2	Rt. 146(N)	See SG-8 (Rt. 146 South Soil) Results					
SG-3	UCR (N)	42.4	2.93	1.65	1.57	7.52	14.5
SG-4	RWW	47.7	3.54	1.24	1.77	8.32	9.3
SG-5	Rt. 107	41.4	3.14	1.80	1.57	8.08	12.6
SG-6	Jamestown	52.5	3.84	1.10	1.92	8.78	NM
SG-7	Charles St.	43.2	3.01	1.80	1.51	7.83	NM
SG-8	Rt. 146(S)	39.9	4.63	1.52	2.32	10.79	10.9

Note: 1. Resilient moduli were determined using M_R test results in Table 3.5.
2. NM – No Material.

Table 3.8 Experimental Design for Rt. 2 Subgrade Soil

Temperature ^{0}F	No. of Blows	Moisture Content (%)		
		OMC -4%	OMC	OMC +2%
30	25	1MR1	1MR2	1MR3
	35	1MR4	1MR5	1MR6
45	25	1MR7	1MR8	1MR9
	35	1MR10	1MR11	1MR12
60	25	1MR13	1MR14	1MR15
	35	1MR16	1MR17	1MR18
75	25	1MR19	1MR20	1MR21
	35	1MR22	1MR23	1MR24

Table 3.9 Experimental Design for Rt. 146 Subgrade Soil

Temperature ^{0}F	No. of Blows	Moisture Content (%)		
		OMC -4%	OMC	OMC +1%
30	25	2MR1	2MR2	2MR3
	35	2MR4	2MR5	2MR6
45	25	2MR7	2MR8	2MR9
	35	2MR10	2MR11	2MR12
60	25	2MR13	2MR14	2MR15
	35	2MR16	2MR17	2MR18
75	25	2MR19	2MR20	2MR21
	35	2MR22	2MR23	2MR24

Table 3.10 Summary of M_R Test Results for Rt. 2 Subgrade Soil

Serial No.	w/c %	Blow	Density pcf	Temp. F	K1	K2	R^2
1MR1	2.7	25	126.3	29.9	7600.1	0.61	0.93
1MR2	6.7	25	131.8	29.9	3989.9	0.47	0.87
1MR3	8.7	25	129.0	30.3	4252.1	0.39	0.76
1MR4	2.7	35	130.3	29.9	2860.7	0.69	0.80
1MR5	6.7	35	132.5	30.1	3510.9	0.61	0.93
1MR6	8.7	35	130.5	29.9	1829.0	0.61	0.51
1MR7	2.7	25	125.0	45.0	4074.4	0.59	0.93
1MR8	6.7	25	131.8	45.1	3077.2	0.61	0.96
1MR9	8.7	25	129.0	45.0	3808.2	0.45	0.86
1MR10	2.7	35	130.3	45.1	8572.7	0.58	0.85
1MR11	6.7	35	131.9	45.1	2445.5	0.63	0.96
1MR12	8.7	35	130.5	45.0	2302.5	0.53	0.85
1MR13	2.7	25	123.8	60.5	7312.4	0.46	0.87
1MR14	6.7	25	126.7	59.6	3136.5	0.57	0.95
1MR15	8.7	25	127.4	60.2	3283.0	0.49	0.81
1MR16	2.7	35	126.0	59.4	5649.1	0.53	0.88
1MR17	6.7	35	127.5	60.5	4522.3	0.52	0.97
1MR18	8.7	35	129.8	59.6	1835.9	0.68	0.95
1MR19	2.7	25	129.9	75.9	12454.0	0.33	0.74
1MR20	6.7	25	131.3	74.8	7338.3	0.49	0.82
1MR21	8.7	25	130.7	75.9	2827.9	0.54	0.89
1MR22	2.7	35	131.8	74.6	8668.2	0.53	0.80
1MR23	6.7	35	130.6	75.6	2730.6	0.58	0.89
1MR24	8.7	35	129.0	74.5	3612.3	0.51	0.90

Table 3.11 Summary of M_R Test Results for Rt. 146 Subgrade Soil

Serial No.	w/c %	Blow	Density pcf	Temp F	K1	K2	R^2
2MR1	1.9	25	129.1	29.9	4425.8	0.60	0.91
2MR2	5.9	25	129.1	29.9	6430.0	0.30	0.74
2MR3	6.9	25	128.3	30.3	5786.7	0.30	0.73
2MR4	1.9	35	132.0	29.9	4007.7	0.57	0.94
2MR5	5.9	35	132.8	30.1	3060.5	0.46	0.88
2MR6	6.9	35	130.4	29.9	5802.2	0.24	0.61
2MR7	1.9	25	129.1	45.0	5262.8	0.55	0.94
2MR8	5.9	25	130.7	45.1	3832.6	0.41	0.88
2MR9	6.9	25	129.8	45.0	2177.1	0.45	0.68
2MR10	1.9	35	132.0	45.1	6407.7	0.51	0.93
2MR11	5.9	35	132.8	45.1	3944.1	0.48	0.91
2MR12	6.9	35	128.3	45.0	2842.1	0.56	0.83
2MR13	1.9	25	127.6	60.5	3563.7	0.61	0.96
2MR14	5.9	25	129.0	59.6	2570.7	0.61	0.95
2MR15	6.9	25	128,7	60.2	2480.9	0.59	0.94
2MR16	1.9	35	130.6	59.4	3743.8	0.61	0.97
2MR17	5.9	35	132.9	60.5	2683.6	0.64	0.93
2MR18	6.9	35	129.4	59.6	2807.6	0.52	0.89
2MR19	1.9	25	127.5	75.9	3342.8	0.63	0.94
2MR20	5.9	25	133.0	74.8	2915.7	0.60	0.98
2MR21	6.9	25	130.3	75.9	2919.0	0.59	0.94
2MR22	1.9	35	131.8	74.6	5662.8	0.54	0.94
2MR23	5.9	35	130.6	75.6	2946.1	0.43	0.86
2MR24	6.9	35	129.0	74.5	2740.7	0.53	0.92

Table 3.12 M_R Prediction Equations from Multi-Regression Analysis

Route 2 Site

(1) Normal Condition (45° – 75°F) $R^2 = 0.75$

$\log M_R = -0.172271 - 0.124976(w/c) - 0.004992T + 0.011569\gamma_d + 0.53287\log\theta + 0.001058T*(w/c)$

(2) Near Frozen Condition (30°F) $R^2 = 0.76$

$\log M_R = 4.381076 - 0.086742(w/c) - 0.101409T + 0.560376\log\theta$

(3) Normal + Near Frozen Condition (30° - 75°F) $R^2 = 0.75$

$\log M_R = -0.206625 - 0.103865(w/c) - 0.002868T + 0.010692\gamma_d + 0.539381\log\theta + 0.000734T*(w/c)$

Route 146 (south) Site

(4) Normal Condition (45° – 75°F) $R^2 = 0.80$

$\log M_R = 2.002999 - 0.120806(w/c) - 0.005312T - 0.006642\gamma_d + 0.540768\log\theta + 0.001024T*(w/c)$

(5) Near Frozen Condition (30°F) $R^2 = 0.90$

$\log M_R = 4.710487 - 0.17782(w/c) - 0.021839T - 0.022542\gamma_d + 0.413686\log\theta + 0.002983T*(w/c)$

(6) Normal + Near Frozen Condition (30° - 75°F) $R^2 = 0.81$

$\log M_R = 1.636171 - 0.100338(w/c) - 0.002485T - 0.004867\gamma_d + 0.508567\log\theta + 0.000721T*(w/c)$

Combined (Rt. 2 and Rt. 146)

(7) Normal Condition (45° – 75°F) $R^2 = 0.67$

$\log M_R = 1.525237 - 0.121892(w/c) - 0.005479T - 0.002765\gamma_d + 0.53209\log\theta + 0.001292T*(w/c)$

(8) Near Frozen Condition (30°F) $R^2 = 0.65$

$\log M_R = 5.686243 - 0.627756(w/c) - 0.093928T - 0.012381\gamma_d + 0.486062\log\theta + 0.017306T*(w/c)$

(9) Normal + Near Frozen Condition (30° - 75°F) $R^2 = 0.66$

$\log M_R = 1.245644 - 0.096421(w/c) - 0.003421T - 0.00153\gamma_d + 0.523523\log\theta + 0.000898T*(w/c)$

Table 3.13 A Sample Calculation of Effective M_R for Rt. 2 Subgrade

Month	Temp F	w/c %	Mri ksi	Relative Damage
January	34.7	7.0	9.85	0.06
February	34.3	9.5	6.73	0.15
March	35.2	9.3	7.01	0.14
April	52.3	8.8	9.01	0.08
May	59.6	8.3	10.21	0.06
June	66.6	8.4	10.81	0.05
July	68.9	8.2	11.22	0.05
August	70.5	7.7	11.83	0.04
September	53.3	7.4	10.65	0.05
October	48.6	8.3	9.22	0.07
November	42.7	7.0	10.39	0.06
December	34.4	6.5	10.60	0.05
	Average Relative Damage		0.073	
	Effective M_R, ksi		9.30	

Note:
1. Bulk Stress = 10.02 psi, Max. Dry Density = 131.1 pcf
2. Prediction Equation (9) in Table 3.12

Table 3.14 A Sample Calculation of Effective M_R for Rt. 146S Subgrade

Month	Temp F	w/c %	Mri ksi	Relative Damage
January	28.7	8.8	7.29	0.13
February	35.9	9.4	7.19	0.13
March	42.7	9.5	7.67	0.11
April	44.0	9.1	8.21	0.10
May	53.2	8.8	9.39	0.07
June	66.3	8.8	10.75	0.05
July	72.8	9.3	11.09	0.05
August	69.1	9.0	10.89	0.05
September	60.0	9.2	9.69	0.07
October	50.9	8.7	9.28	0.07
November	43.3	8.3	9.06	0.08
December	39.0	8.4	8.58	0.09
	Average Relative Damage		0.084	
	Effective M_R, ksi		8.78	

Note:
1. Bulk Stress = 10.79 psi, Max. Dry Density = 133.1 pcf
2. Prediction Equation (9) in Table 3.12

Table 3.15 Effective Resilient Modulus of Each Site

Site	Soil Type	Bulk Stress psi	Dry Density pcf	Avg. Relative Damage	Effective Mr (psi)
Rt. 2S	A-1-b	10.02	131.1	0.07	9,304
Rt. 146N	A-1-b	See Rt. 146S Soil Results			
UCR	A-1-b	7.52	132.0	0.10	7,982
RWW	A-1-b	8.32	121.5	0.09	8,733
Rt. 107	A-1-b	8.08	134.7	0.12	7,506
Jamestown	A-1-b	8.78	127.5	0.08	8,795
Charles St.	A-1-b	7.83	122.4	0.14	7,711
Rt. 146S	A-1-b	10.79	133.1	0.19	8,782
				Average	8,402
				S.D.	669

Note: 1. Bulk Stress is at ADSS
2. Effective M_R using the prediction equation (9) in Table 3.12.

Figure 3.1 Site Location Map

CHAPTER 4 ESTIMATION OF EQUIVALENT SINGLE AXLE LOADS (ESALs) IN RHODE ISLAND

4.1 Introduction

One of essential parameters for pavement design is cumulative 18-kip equivalent single axle loads (ESALs). The 1993 AASHTO Guide for Design of Pavement Structures (AASHTO Guide) includes a procedure to convert traffic data of different axle loads into 18-kip ESALs ("Guide" 1993). Then the cumulative 18-kip ESALs can be calculated over a certain analysis period and can be presented in the form of a graph to show the ESAL progression over time.

According to the AASHTO Guide, the four major considerations that influence the accuracy of traffic estimates and the life cycle of pavement are: (1) the correctness of the load equivalency values, (2) the accuracy of traffic volume and weight information, (3) the prediction of ESALs over the design period, and (4) the changes in Present Serviceability Index (PSI). Every state collects the data for traffic weight based upon the functional classification of road. This is accumulated in the format of the Federal Highway Administration (FHWA) W-4 truck weight tables, which are tabulations of the number of axles observed within a series of load groups with each load group covering a 2-kip interval. Traffic information relative to truck type, i.e., axle configuration, is provided in W-2 tabulations (distribution of vehicles counted and weighed).

Since pavements, new or rehabilitated, are usually designed for periods ranging from 10 years to 20 years or more; it is necessary to predict the ESALs for this period of time, i.e., the performance period. The performance period, often referred to as the design period, is defined as the period of time that an initial (or rehabilitated) structure will last before reaching its terminal serviceability (p_t). Any performance period may be used with the AASHTO Guide, since design is based on the total number of equivalent single axle loads. However, experience may indicate a practical upper limit based on considerations other than traffic. The ESALs for the performance period represent the cumulative number from the time the roadway is opened to traffic to the time when the serviceability is reduced to a terminal value, i.e., p_t. If the traffic is underestimated, the actual time to p_t will probably be less than the predicted performance period, thereby resulting in increased maintenance and rehabilitation costs.

The equivalent loads derived from many traffic prediction procedures represent the totals for all lanes in both directions of travel. This traffic must be distributed by assigning 50% of the traffic to each direction, unless available measured traffic data warrant some other distribution. In regard to lane distribution, the traffic in one direction is assigned to each of the lanes in that direction for purposes of structural design, if measured distributions are not available. Some states have developed lane distribution factors for facilities with more than one lane in a given direction. These factors vary from 60% to 100% of the one-directional traffic, depending on the total number of lanes in the facility.

Predictions of future traffic are often based on past traffic history. Several factors can influence such predictions. Traffic may remain constant, or increase linearly or at an accelerating (exponential) rate. In most cases, highways classified as principal arterial or interstate will have exponential growth (comparable to compound interest on investments). Traffic on some minor arterial or collector- type highways may increase linearly, while traffic on some residential streets may not change because the use remains constant. Thus, the designer must make provision for growth in traffic from the time of the last traffic count or weighing through the performance period selected for the project under consideration. Appendix B provides appropriate information for estimating future traffic growth based on an exponential growth rate. If zero or negative growth traffic is anticipated, a zero or negative growth factor can be used. In most cases, appropriate growth factors can be selected from the table shown in Appendix D of AASHTO Guide (“Guide” 1993).

The load equivalency factor increases approximately as a function of the ratio of any given axle load to the standard 18-kip single load raised to the fourth power. For example, the load equivalency of a 12-kip single axle is given as 0.19, while the load equivalency for 20-kip single axle is 1.56 ("Guide" 1993). Thus, the 20-kip load is 8 times as damaging as the 12-kip load, i.e., $(20/12)^4$. This relationship varies depending on the structural number and terminal serviceability; however, it is generally indicative of load effects. Thus, it is especially important to obtain reliable truck weight information for each truck class and especially for the multi-axle trucks since these vehicles will constitute a high percentage of the total ESALs on most projects.

This chapter illustrates how to estimate the 18-kip ESALs for given sections of Route 2 and Route 146 in Rhode Island.

4.2 Calculating ESALs Applications

To use the AASHTO Guide for the pavement design, the mixed traffic must be converted to an equivalent number of 18-kip single axle load. The procedure for accomplishing this conversion includes:

(1) derivation of load equivalency factors,

(2) conversion of mixed traffic to 18-kip ESAL applications, and

(3) direction and lane distribution considerations.

Load equivalency factors represent the ratio of the number of repetitions of any axle load and axle configuration (single, tandem, tridem) necessary to cause the same reduction in PSI as an application of an 18-kip single axle load. These load equivalency factors for flexible pavements in AASHTO Guide were determined using the following equations:

$$\log_{10}\left[\frac{W_{tx}}{W_{t18}}\right] - 4.78\times\log_{10}(18+1) - 4.79\times\log_{10}(L_x+L_2) + 4.33\times\log_{10}L_2 + \left[\frac{G_t}{\beta_x}\right] - \left[\frac{G_t}{\beta_{18}}\right]$$

$$G_t = \log_{10}\left[\frac{4.2 - p_t}{4.2 - 1.5}\right]$$

$$\beta_x = 0.40 + \left[\frac{0.081 \times (L_x + L_2)^{3.23}}{(SN+1)^{5.19} \times L_2^{3.23}}\right]$$

where

L_x = load on single axle or one tandem axle set (kips),

L_2 = axle code (1 for single axle and 2 for tandem axle),

SN = structural number,

p_t = terminal serviceability, and

β_{18} = value of β_x when L_x is equal to 18 and L_2 is equal to 1.

When calculating ESALs for the design of a particular project, it is convenient to convert the estimated traffic distribution into truck load factors. Appendix D of AASHTO Guide provides two methods to calculate truck load factors from W-4 tables: (1) where axle load information is available from a weigh station, the truck load factor can be calculated directly; and (2) when information is not available directly from weigh station, the factor can be calculated using representative values for each various truck classification. AASHTO Guide also provides a work sheet for either method to calculate ESALs using truck load factors.

4.3 ESAL Calculation for Rt. 2 and Rt. 146

Table 4.1 and Table 4.2 show the calculations of 18-kip ESAL's for Rt. 2, a minor arterial rural highway and Rt. 146, a principal arterial rural highway respectively. The percent growth per year for Rt. 2 is 1.25% and for Rt. 146 is 1.31% ("Rhode" 1998). Using these growth rates the growth factors were determined using the tables given in AASHTO Guide.

RIDOT calculates average ESALs data from WIM stations for only vehicle classifications between 4 to 13. RIDOT neglects classification 1 to 3 vehicles due to their light weight. Therefore, from Table 4.1 and 4.2, the design ESALs were determined using only from vehicle classification from 4 to 13.

The first (A) and second (B) columns show the FHWA vehicle classification and type respectively. The third column (C) represents the base year daily volume counts of each vehicle type taken from data collected at classification count stations representative of the design location. Traffic count data and average ESAL based on vehicle classification were obtained from RIDOT Planning and Traffic Management Division. For both sites there were no current data obtained from Weigh-In-Motion (WIM) station. For Rt. 2, the WIM study was done using portable WIM stations, and the most recent traffic data was for the year 1990. For Rt. 146, the data was taken from the permanent WIM in 1996. The traffic count during this period was taken and projected to the year 1998 which is shown in Column (C). The fourth column (D) indicates the growth factor. The fifth column (E) is basically a product of the first two columns with the Rt. 2 values multiplied by 365 to convert from daily to annual traffic. The result is the accumulated applications of specific vehicle types during the analysis period. The sixth column (F) indicates the individual ESAL factor for each of the vehicle types. Unfortunately, the ESAL factor furnished by RIDOT were found to be not reasonable. Hence the URI research team has estimated these values as shown in Table 4.3 based upon their vehicle classification shown in Figure 4.1. The axle loads were determined using Figure 4.1 and 4.2 (Lee et al. 1990). The seventh column (G) is an extension of columns (E) and (F) indicating the total ESAL's (by vehicle type) that might be applied to the sample section during the analysis period.

The summation of these values then is the total 18-kip ESAL traffic that should be used for pavement structural design. Figures 4.3 and 4.4 are plots of the cumulative 18-kip ESAL traffic over the 20-year analysis period for Rt. 2 and Rt.146, respectively. The curve and equation for future traffic (w_{18}) are reflective of the exponential growth rate.

Table 4.3 Comparison of ESAL factors for Rt. 2 and Rt. 146 site along with Estimated Values

Vehicle Classification	Rt. 2	Rt. 146	Estimated Value
4	1.67	1.33	1.67
5	0.61	0.44	1.67
6	1.42	1.68	1.28
7	*	6.07	1.52
8	0.70	2.24	2.86
9	1.15	4.28	2.24
10	*	4.42	1.95
11	0.0185	4.26	6.09
12	*	2.43	5.67
13	*	10.84	5.26

* No data.

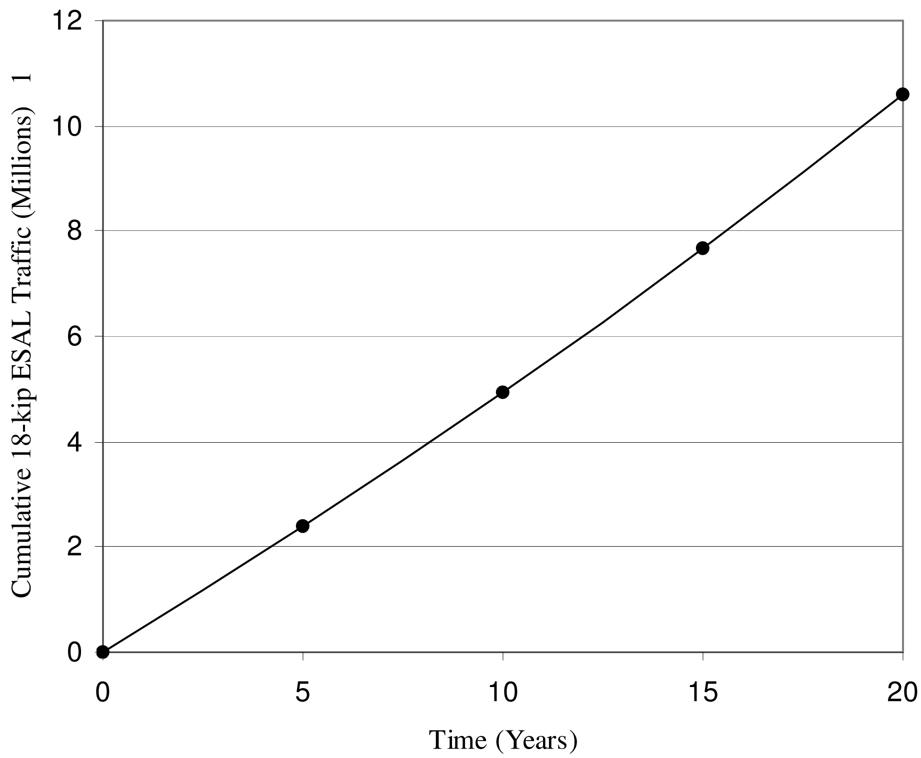

Figure 4.3 Plot of Cumulative 18-kip ESAL Traffic Versus Time for Rt.2 Pavement

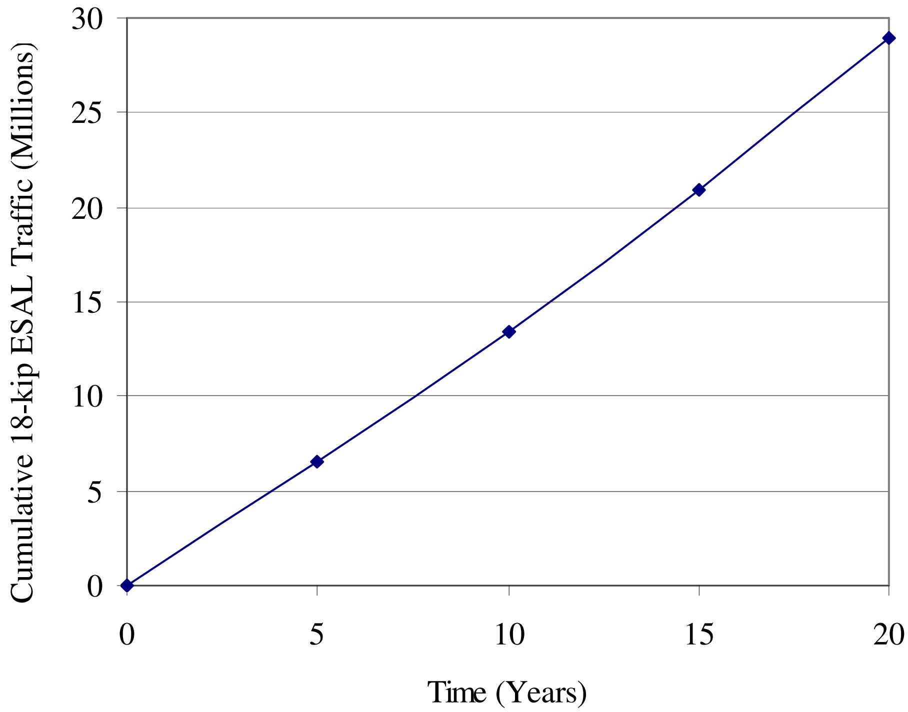

Figure 4.4 Plot of Cumulative 18-kip ESAL Traffic Versus Time for Rt.146 Pavement

CHAPTER 5 ENVIRONMENTAL EFFECTS AND DETERMINATION OF INITIAL STRUCTURAL NUMBER

In addition to the effective roadbed soil resilient modulus and estimated total 18-kip equivalent single axle load (ESAL) applications, other parameters need to be estimated when determining the structural number (SN). This chapter illustrates how to determine the SN for flexible pavement structures of Rt. 2 and Rt. 146 using parameters associated with time constraints, reliability, environmental impacts and serviceability loss.

5.1 Time Constraints

The analysis period selected for this design example is 20 years. The maximum performance period selected for the initial flexible pavement structure in this example is 15 years. Thus, it was necessary to consider stage construction (i.e., planned rehabilitation) alternatives to develop design strategies that would ensure the minimum acceptable present serviceability index (PSI) is maintained over the analysis period.

5.2 Traffic

Based on average daily traffic and axle weight data, the estimated traffic during the first year (in the design lane) were estimated as 467,576 and 1,266,739 18-kip ESAL applications for Rt. 2 and Rt. 146, respectively.

5.3 Reliability

Due to the Rt. 2 and Rt. 146 roadway classifications a 90-percent overall reliability level was selected for the designs. This means that for a two-stage strategy (initial pavement plus overlay), the design reliability for each stage must be $0.90^{1/2}$ or 95 percent. Similarly, for a three stage strategy (initial pavement plus two overlays), the design reliability for all three stages must be $0.90^{1/3}$ or 96.5 percent.

Another criteria required for the consideration of reliability is the overall standard deviation (S_o). An approximate value of 0.35 was used for the purposes for this illustrative design example.

5.4 Environmental Impacts

The two environmental impacts that affect the pavement life are swelling and frost-heave. The swell rate constant is a factor that can be obtained using the procedure adopted in the Table G.1 of 1993 AASHTO Guide. Since Rhode Island soils are sand or silty sand, i.e., almost non-plastic, the swelling was not considered in this example.

The accumulation of water within the larger soil, when frozen, causes soil expansion and frost heaving beneath the pavement. This results in the formation of continuous ice lenses, layers, veins, or other ice masses. The growth of such distinct bodies of ice is termed as ice segregation.

In Rhode Island there is a great variation of temperature, and roadbed soils can be susceptible to frost heave effects. In order to take these environmental impacts into consideration, a curve has to be developed which correlates the serviceability loss due to frost heave and time. To generate the frost heave curve, it is necessary to estimate three factors: (1) frost heave rate based on the type of roadbed material and its gradation, (2) maximum potential serviceability loss due to frost heave which is based on the drainage quality and depth of frost penetration, and (3) frost heave probability based on past experience.

Due to insufficient data available for evaluating the amount of frost-heave in Rhode Island, the serviceability loss curve from 1993 AASHTO Guide was used (Figure 5.1). It is also assumed that serviceability loss would be same for both Rt. 2 and Rt. 146 due to their relative proximity to each other.

The values for the three frost heave factors from 1993 AASHTO Guide that were used are as follows:

Frost heave rate = 5 mm/day

Frost heave probability = 30%

Maximum present serviceability loss = 2.0

The equation for serviceability loss, that was used to generate frost heave serviceability loss curve (Figure 5.1) is as follows:

$$\Delta PSI = 0.01 \times P_F \times \Delta PSI_{MAX}\left[1 - e^{-(0.02 \times \Phi \times t}\right]$$

5.5 Serviceability Loss

Based on the traffic volume and functional classification of the facility, a terminal serviceability (p_t) of 2.5 was selected. Past experience indicates that the initial serviceability (p_0) normally achieved for flexible pavements in the state is significantly higher than that at the AASHO Road Test (4.6 compared to 4.2). Thus, the overall design serviceability loss for this problem is:

$$\Delta \text{PSI} = p_0 - p_t = 4.6 - 2.5 = 2.1$$

5.6 Effective Roadbed Soil Resilient Modulus

The results that were obtained from Chapter 3 are used in this design example. Thus, the effective roadbed soil resilient moduli were 9,306 psi and 8,783 psi for Rt. 2 and Rt. 146, respectively.

5.7 Development of Initial Stage of a Design Alternative

The strategy with the maximum recommended initial structural number is determined using the effective roadbed soil resilient modulus of 9,304 psi, a two stage reliability of 95 percent, an overall standard deviation of 0.35, a design serviceability loss of 2.1 and the cumulative traffic at the maximum performance period, 7,663,570 18-kip ESAL for Rt. 2

pavement structures. Applying these parameter values to the AASHTO nomograph (Figure 5.2), the result is a maximum initial structural number (SN) of 4.4. Because of serviceability loss due to frost heave however, an overlay will be required before the end of the 15-year design performance period. Using the step-by-step procedure shown in Table 5.1 the service life that can actually be expected is about 12 years. Thus, the overlay that must be designed will need to carry the remaining ESAL traffic over the last 8 years of the analysis period. The w_{18} value of 6.5×10^6 and ΔPSI_{TR} equal to 1.68, would be used to determine the SN required for Rt. 2 pavement structure.

Similarly, the maximum initial SN was determined as 5.1 for Rt. 146 pavement structure using the effective roadbed soil resilient modulus of 8,782 psi, a two stage reliability of 95 percent, an overall standard deviation of 0.35, a design serviceability loss of 2.1 and the cumulative traffic of 20,901,189. However, an overlay will be required before the end of the 15-design performance period, because of serviceability loss due to frost heave. Using the step-by-step procedure as shown in Table 5.2, the service life that can be actually expected is about 13 years. Again, w_{18} value of 17.0×10^6 and ΔPSI_{TR} equal to 1.66 would be used to determine structural number for the Rt. 146 pavement structure.

Table 5.1 Reduction in Performance Period (Service Life) of Initial Pavement Arising From Frost Heave Considerations for Rt. 2.

Initial SN 4.4

Maximum Possible Performance Period (years) 15

Design Serviceability Loss, Δ PSI = $p_0 - p_t$ = 4.6 - 2.5 = 2.1

(1) TIteration No.	(2) Trial Performance Period (years)	(3) Serviceability Loss Due to Frost Heave Δ PSI_{FH}	(4) Corresponding Serviceability Loss Due to Traffic Δ PSI_{TR}	(5) Allowable Cumulative Traffic (18-kip ESAL)	(6) Corresponding Performance Period (years)
1	13	0.44	1.66	6.0×10^6	11.8
2	12.4	0.42	1.68	6.5×10^6	12.5

Table 5.2 Reduction in Performance Period (Service Life) of Initial Pavement Arising From Frost Heave Considerations for Rt. 146.

Initial SN <u>5.1</u>

Maximum Possible Performance Period (years) <u>15</u>

Design Serviceability Loss, Δ PSI = $p_0 - p_t$ = <u>4.6 - 2.5 = 2.1</u>

(1) Iteration No.	(2) Trial Performance Period (years)	(3) Serviceability Loss Due to Frost Heave ΔPSI_{FH}	(4) Corresponding Serviceability Loss Due to Traffic ΔPSI_{TR}	(5) Allowable Cumulative Traffic (18-kip ESAL)	(6 Corresp Perfor Per (ye
1	13	0.44	1.66	17.0×10^6	12

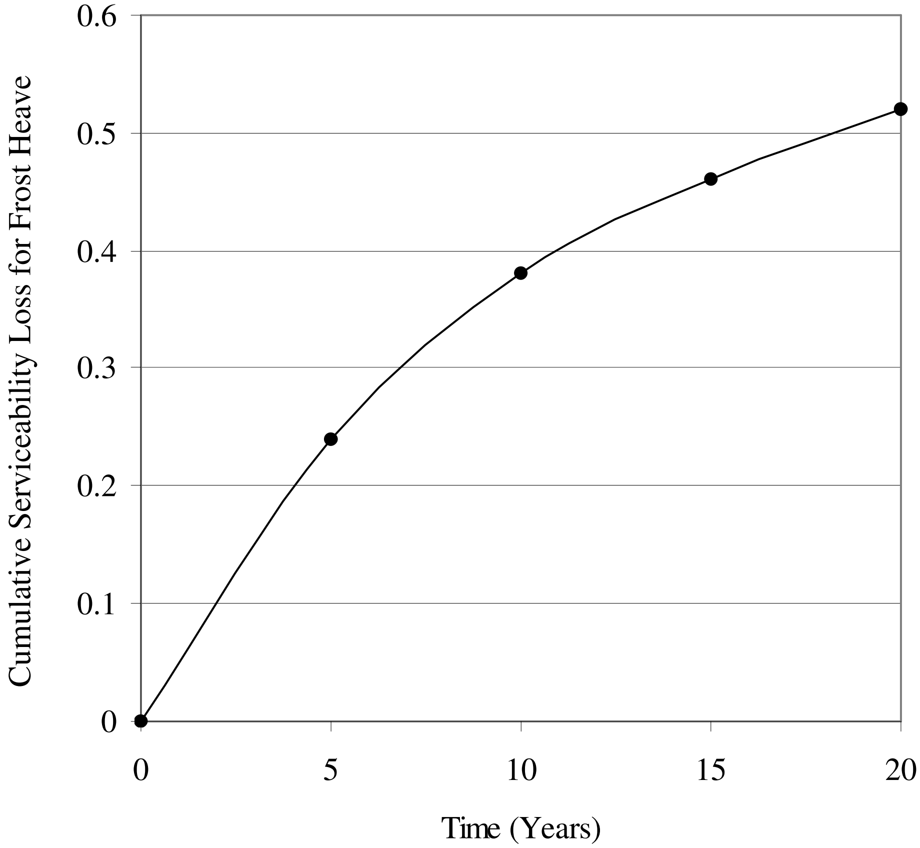

Figure 5.1 Plot of Environmental Serviceability Loss Versus
Time for Frost-Heave Conditions Considered

CHAPTER 6 ESTIMATION OF LAYER COEFFICIENTS

6.1. Introduction

The layer coefficient is used in the AASHTO Guide to determine the thickness of the layers of flexible pavements. A layer coefficient is needed for each pavement layer: typically asphalt surface, asphalt base or granular base, and granular subbase. However, RIDOT currently uses a combination of an asphalt surface and asphalt base, i.e., deep strength pavement structure. Therefore, the structural number (SN) equation for the Rhode Island flexible pavement structure is:

$$SN = a_1 D_1 + a_3 D_3 m_3 \quad (6.1)$$

where

a_1 and a_3 are layer coefficients of combined asphalt layer and granular subbase, respectively,

D_1 and D_3 are layer thickness of combined asphalt layer and granular subbase, respectively,

a_1 D_1 represent the SN values for asphalt surface and base,

m_3 represents the drainage coefficient for subbase material, and

a_3 D_3 represent the SN values for granular subbase material.

The layer coefficient is an indicator of strength, which is dependent upon the resilient modulus (E_{AC} or E_{SB}). It determines the required thickness of each layer. If the modulus decreases, the layer coefficient also decreases. However it may be noted that the thickness of the each layer increases, as the layer coefficient decreases.

6.2. Layer Coefficients for Unbound Layer Materials

Conventionally, two predominant methods have been used to estimate layer coefficients (1) Direct Method, and (2) Relative Method.

6.2.1. Direct Method

In the direct method, the layer coefficient, a_i, is related to a strength parameter, such as, the resilient modulus, as shown in the following equation:

$$a_i = A * M_R^{B} \tag{6.2}$$

Where, A, B = experimentally derived constants, and

M_R = resilient modulus (AASHTO T274-82)

6.2.2. Relative Method

Instead of using a strength parameter, a "known" layer coefficient, a_{ref}, is used to obtain the "unknown" layer coefficient, a_i, for example:

$$a_i = a_{ref} * [\frac{M_{Ri}}{M_{ref}}]^{B} \tag{6.3}$$

$$a_i = a_{ref} * [\frac{M_{Ri}}{M_{ref}}]^{B} = \left[\frac{a_{ref}}{M_{refi}}\right] * M_{Ri} \wedge B = A * M_R^{B} \tag{6.4}$$

By studying both equations carefully, it can be seen that the two methods may be considered identical, since a_{ref} and M_{Rref} are constants in the relationships.

6.3. Determination of Subbase Layer Coefficients

In this study the direct method, was used, and the layer coefficient was estimated using the following equation (Rada and Witczak, 1981):

$$a_3 = 0.227 (\log_{10} E_{SB}) - 0.839 \tag{6.5}$$

where

E_{SB} = resilient modulus, psi (Rada and Witzak 1981).

Resilient Modulus tests for subbase materials were performed in accordance with the procedure of the AASHTO T292-91, and results are summarized in Table 6.1. After determining resilient moduli at the mid-depth of subbase layers, the layer coefficients were estimated as shown in Table 6.2.

6.4. Determination of Layer Coefficients for Bound Layers

One objective of this study was to realistically estimate layer coefficients of bound layers used for construction of flexible pavement structures in Rhode Island. This section presents test results for the laboratory prepared specimens, and determination of layer coefficients of hot mix asphalt (HMA).

In order to determine the layer coefficients of bound layers, i.e., surface, modified binder, and modified base, the Optimum Binder Content (OBC) must first be determined. The results of the Marshall mix design are based on the RIDOT specifications ("Standard" 1997). Since the specifications require that the air voids should fall between 3-5%, for Class I-1, the OBC was determined at 4% air void. For the modified binder and base layers the specifications require that the air voids should be between 3-8%. Therefore, the OBC was obtained at 5.5% air voids. Therefore, Marshall mix designs were performed for HMAs from eight sites during this study (Lee et al. 1994). The details of mix designs are included in the Appendix C.

HMA specimens prepared with corresponding OBCs were tested for resilient modulus (E_{AC}), and layer coefficients were estimated for eight typical Rhode Island pavement structures. The sites selected encompass the five major contractors, who are producing HMA in Rhode Island. The AASHTO Guide provides a chart to estimate layer coefficients for asphalt concrete by using the resilient modulus values at 68°F (Figure 2.1 of the 1993 AASHTO Guide). Based upon the laboratory test results, layer coefficients were estimated for the design of flexible pavement structures in Rhode Island. Test results and estimated layer coefficients are summarized in Table 6.3.

Table 6.1 Resilient Modulus Test Results for Subbase Materials in Accordance with AASHTO T292-91

ID No.	w/c (%)	OMC	Comp-action Blow	Density (pcf)	Temp. C	E_{SB} k_1	k_2	R^2
SB-1	6.7	7.5	25	129.7	22.4	5808.2	0.37	0.84
SB-2	5.1	6.1	25	128.5	23.8	4126.4	0.56	0.9
SB-5	5.3	6	25	124.7	20.4	4054.7	0.5	0.94
SB-7	6.9	7.2	25	131.3	16	8479.5	0.34	0.74
SB-8	5.7	6.6	25	135.2	19.6	5948.1	0.36	0.88

Note: 1. $E_{SB} = K_1\ \theta \wedge K_2$

2. R^2 = coefficient of determination

Table 6.2 Layer Coefficients for Subbase Materials

Site	Soil Type	Specimen ID	E_{SB} (psi)	Layer Coefficient a_3
Rt. 2	A-1-b	SB-1	13,620	0.1
Rt. 146	A-1-a	SB-2	13,185	0.1
UCR	A-1-b	**SB-3**	NM	-
RWW	A-1-b	SB-4	NM	-
Rt. 107	A-1-b	SB-5	12,069	0.09
Jamestown	A-1-b	SB-6	NM	-
Charles Street	A-1-b	SB-7	18,539	0.13
Rt. 146 s	A-1-b	SB-8	12,143	0.09
		Average	13,911	0.10

Note: 1. $a_3 = 0.227 \log_{10}(E_{SB}) - 0.839$

2. NM stands for no materials.

Table 6.3 Results of Resilient Modulus Test and Estimated Layer Coefficients

ID #	Site	Layer	E_{AC} (ksi)	a_i
PH I-1 D'Ambra	Rt. 2	CI-1	325	.37
		MBi	540	0.47
		MBA	480	0.45
Ph I-2 T.G.	Rt. 146N	CI-1	433	0.43
		MBa	437	0.43
Ph I-3 Lynch	UCR	CI-1	411	0.42
		MBi	372	0.40
		MBa	497	0.46
Ph II-1 T.G.	RWW	CI-1	308	0.37
		MBi	371	0.40
		MBa	303	0.36
PhII-2 Lynch	Rt. 107	CI-1	318	0.37
		MBi	410	0.42
		MBa	372	0.40
Ph II-3 Cardi	Rt. 138 Jamestown	CI-1	312	0.37
		MBi	315	0.36
		MBa	381	0.42

Ph II-4 D'Ambra	Charles St.	CI-1	291	0.36
		MBi	372	0.40
		MBa	392	0.42
Ph II-5 Forte	Rt. 146 s	CI-1	247	0.34
		MBa	384	0.42

Chapter 7 DRAINAGE COEFFICIENTS

7.1 Introduction

Added moisture will result in a loss of stiffness for all unbound aggregate materials. Reductions in modulus values of more than 50 percent have been reported (Finn et al. 1977; Rada and Witzak 1981). However, the AASHTO Interim Guide did not fully recognize the effect of positive drainage within the pavement structure on the life of the pavement. The AASHTO Guide (1993) introduced drainage coefficients to account for this important factor. The "m" value is used in flexible pavement design to reduce or increase the layer coefficient of the granular base and subbase layers.

The general definitions corresponding to the different drainage levels from the pavement structure are summarized in Table 7.1. For comparison purposes, the drainage conditions at the AASHTO Road Test are considered to be fair, i.e., water was removed within 1 week.

The AASHTO Guide recommended m_i values as a function of the quality of drainage and the percent of time during the year of which the pavement structure is normally exposed to moisture levels approaching saturation as shown in Table 7.2. Obviously, the latter is dependent on the average yearly rainfall and the prevailing drainage conditions. As a basis for comparison, the m_i values for conditions at the AASHTO Road Test is 1.0, regardless of the type of material.

It is important to note that the values apply only to the effects of drainage on untreated base and subbase layers. Although improved drainage is certainly beneficial to stabilized or treated materials, the effects on performance of flexible pavements are not as profound as those quantified above.

7.2 Estimation of Drainage Coefficients for Flexible Pavement Structures

Drainage is generally treated by considering the effect of water on the properties of the pavement layers and the consequences to the structural capacity of the pavement.

Appendix DD of the AASHTO Guide describes the development of drainage coefficients used in pavement design procedures.

The drainage conditions must be assessed. The method recommended by the FHWA Report TS-80-224 ("Highway" 1980) requires the calculation of the time required to drain the base layer to 50 percent saturation (T_{50}). It is determined for different combinations of permeability (k), length of drainage path (L), effective porosity (η_e), and slope ($\tan\alpha$).

The coefficient of permeability was calculated using the following equation:

$$k = \frac{QL}{Ath}$$

where

k = coefficient of permeability, cm/sec,

Q = discharge of water in t seconds, cm^3,

L = length of the specimen, cm,

A = area of the specimen's cross section, cm^2,

t = time to discharge Q water, seconds, and

h = pressure head, cm of water.

The measurement of subsurface drainage is generally based on the time required for 50 percent of the unbound water to be removed from the layer to be drained. The Casagrande flow equation for estimating the 50-percent drainage time is expressed as:

$$t_{50} = \frac{\eta_e \times L^2}{2 \times k \times \left(H + L \times \tan\alpha\right)}$$

where

t_{50} = time for 50 percent of unbound water to drain (days),

η_e = effective porosity (80 percent of absolute porosity),

L = length of flow path (feet),

k = permeability constant (ft./day),

H = subbase thickness (feet), and

$\tan\alpha$ = slope of the base layer

The results of these calculations are shown in Table 7.3 as an example. Using the information provided by Moultan (1980), the quality level was established as shown in Table 7.1. The present study estimated the quality of drainage coefficient by correlating this table with the permeability of the materials tested in feet per day.

The drainage quality along with the percentage of time the pavement structure is at or near saturation enables the designer to use Table 7.2. However, to estimate the drainage coefficients for other materials, both parameters need to be determined by a step-by-step procedure.

7.2.1 Laboratory Testing

In the present study, the permeability was used to estimate the quality of drainage. The coefficient of permeability was determined in accordance with AASHTO T215-70 Permeability of Granular Soils (Constant Head). A permeability test for subbase was performed using a permeameter, which is capable of performing either a constant head or a falling head test. The constant head test was used for each material at the optimum moisture content (OMC) and maximum dry density (γ_{max}) in the present study. It is a constant head method for the laminar flow of water goes through granular soils. It is intended to establish representative values of the coefficient of permeability for granular materials used as subbase courses. Materials should not have more than 10 percent passing the 0.075 mm (No. 200) sieve.

The procedure of AASHTO T215-70 requires a 150 mm (6 in.) mold when the maximum particle size lies between sieve openings 9.5 mm (3/8 in.) and 19 mm (3/4 in.) and more than 35 percent of the total soil is retained on the 9.5 mm (3/8 in.) sieve. The minimum diameter of the permeameter should be 8 x maximum particle size (8 x .75 in. = 6 in.). A Soil Test Model K-612A compaction permeameter was used in the present study. These mold and base permit mechanical compaction at OMC prior to testing. The same Soil Test mechanical compactor as proctor testing was used to prepare specimens.

7.2.2 Test Results and Analysis

The permeability test results of the different sites in Rhode Island subbase are summarized in Table 7.4, and the drainage quality as well as the coefficient of drainage are provided in Table 7.5. The permeability for the subbase materials ranged from 0.765 to 0.9938 ft/day, while those of the cold recycled base courses had a higher range.

The drainage quality of the material was fair, for both cut and fill sections. The classifications for the drainage qualities are based on the Table 7.1. In order to choose a drainage quality, the permeability, thickness of the subbase, the slope, and the length of the drainage path must be known. Based on the amount of time (in days) it takes to drain the layer to 50 percent saturation (damp), the drainage quality can be determined. An example to estimate drainage coefficient is shown below.

Example The Coefficient of Drainage for Rt. 146 South Subbase Material.

Given: Rt. 146 South subbase material

Subbase Thickness, H = 12 inches = 1 foot

Drainage Path, L = 24 feet

Slope range, tanα = 0.02

Determine: The coefficient of drainage, **m**.

Solution:

- Result of the permeability testing (from Table 7.5), k = 0.850 ft/day
- The permeability k = 0.850 ft/day lies between 0.1 and 1.0 in Table 7.3, thus it is necessary to interpolate.
- It will take 9 days to drain Rt. 146 subbase material with a 0.02 slope. Drainage quality is fair, from the second part of Table 7.1
- The percent of time pavement structure is exposed to moisture levels approaching saturation is 5 – 25 %.
- The coefficient of drainage for Rt. 146 South is 0.9 from Table 7.2.

Table 7.1 Quality of Drainage

Quality of Drainage	Calculated Drainage Time	Water Removed Within
Excellent	2-4 hours	2 hours
Good	½ to 1 day	1 day
Fair	3 to 6 days	1 week
Poor	18-36 days	1 month
Very Poor	More than 36 days	(water will not drain)

Source: 1993 AASHTO Guide for Design of Pavement Structures

Table 7.2 Recommended m_i Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	Less Than 1 %	1-5 %	5-25 %	Greater than 25 %
Excellent	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very Poor	1.05-0.95	0.95-0.75	0.75-0.40	0.40

Source: 1993 AASHTO Guide for Design of Pavement Structures

Table 7.3 Time (in days) to drain base layer to 50 percent saturation (damp)

Permeability k, (ft/day)	Porosity, η_e	Slope, $\tan\alpha$	H = 1		H = 2	
			L = 12	L = 24	L = 12	L = 24
0.1	0.015	0.01	10	36	6	20
		0.02	9	29	5	18
1.0	0.027	0.01	2	6	5	18
		0.02	2	5	1	3
10.0	0.048	0.01	0.3	1	0.2	0.6
		0.02	0.3	1	0.2	0.6
100.0	0.08	0.01	0.05	0.2	0.03	0.1
		0.02	0.05	0.2	0.03	0.1

Source: Appendix DD of 1986 AASHTO Guide

Table 7.4 Results of Permeability Test

Material	Maximum Dry Density γ_{max} (pcf)	Density γ (pcf)	Void Ratio e	Effective Porosity $\boxed{0.80 \times \frac{e}{(1+e)}}$ η_e	Flow cm^3	Permeability k, ft/day (cm/s)
Rt. 2	129.7	126.7	0.33	0.198	934	0.912 ($3.2x10^{-4}$)
Rt. 146N	See Rt. 146S Results					
Upper College Road	N/A					
Roger William's Way	127.0	128.6	0.31	0.192	450	0.828 ($2.9x10^{-4}$)
Rt. 107	124.7	128.0	0.32	0.194	670	0.858 ($3x10^{-4}$)
Jamestown	131.2	134.3	0.25	0.160	700	0.938 ($3.3x10^{-4}$)
Charles St.	131.3	131.4	0.28	0.175	460	0.765 ($2.7x10^{-4}$)
Rt. 146S	128.5	126.7	0.33	0.198	900	0.850 ($3x10^{-4}$)
Cold Recycled Base Course						
Rt. 102	123.5	124.0	0.36	0.223	900	1.228 ($4.3x10^{-4}$)
Rt. 116	127.0	126.8	0.33	0.198	980	3.315 ($1.2x10^{-3}$)

Note: N/A stands for not available.

Table 7.5 Drainage Coefficient Based on Drainage Quality

Material	Perm. k, (ft/day)	Slope Tan α	Layer Thickness H (feet)	Path Length L (feet)	Water Removed Within (days)1	Quality of Drainage 2	Estimated Drainage Coefficient (m_i)
Rt. 2	0.912	0.02	1	24	7	Fair	0.9
Rt. 146 N	See Rt. 146S Results						
Upper College Road	N/A						
Roger William's Way	0.828	0.02	1	24	9	Fair	0.90
Rt. 107	0.858	0.02	1	24	9	Fair	0.90
Jamestown	0.938	0.02	1	24	7	Fair	0.90
Charles St.	0.765	0.02	1	24	10	Fair	0.80
Rt. 146 S	0.850	0.02	1	24	9	Fair	0.90
Cold Recycled Base Course							
Rt. 102	1.228	0.02	1	24	5	Fair	1.0
Rt.116	3.315	0.02	1	24	4	Fair	1.0

1 From Table 7.3
2 From Table 7.1

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CHAPTER 8 APPLICATION OF DEVELOPED PARAMETERS TO THE AASHTO GUIDE AND DARWin

The effectiveness of the developed parameter values was evaluated through pavement structures of Rt. 2 and Rt. 146 using the 1993 AASHTO Guide and DARWin 2.01.

8.1 Pavement Layer Materials Characterization

The moduli for each, determined using the recommended laboratory test procedures, are as follows:

Rt. 2

Asphalt Concrete: Class I-1 = 325,000 psi

Modified Binder = 540,000 psi

Modified Base = 480,000 psi

Granular Subbase: E_{SB} = 13,600 psi

Rt. 146

Asphalt Concrete: Class I-1 = 433,000 psi

Modified Base = 437,000 psi

Granular Subbase: E_{SB} = 12,100 psi

8.2 Layer Coefficients

The structural layer coefficients (a_i-values) corresponding to the moduli reported in the previous section are as follows:

Rt. 2

Asphalt Concrete: $a_1 = 0.45$ (weighted)

Granular Subbase: $a_3 = 0.10$

Rt. 146

Asphalt Concrete: $a_1 = 0.39$ (weighted)

Granular Subbase: $a_3 = 0.09$

8.3 Drainage Coefficient

Since the base in Rhode Island is asphalt base, only the granular subbase has a drainage coefficient value. The drainage coefficient of subbase for Rt. 2 and Rt. 146 was 0.90.

8.4 Determination of Structural Layer Thicknesses for Initial Structure

It should be recognized that, for flexible pavements, the structure is a layered system and should be designed accordingly. First, the structural number required over the roadbed soil should be computed. In the same way, the structural number required over the subbase layer and the base layer should also be computed, using the applicable strength values for each. By working with differences between the computed structural numbers required over each layer, the minimum allowable thickness of any given layer can be computed. For example, the minimum allowable structural number for the subbase material would be equal to the structural number required over the subbase subtracted from the structural number required over the roadbed soil. In a like manner, the structural numbers of the other layers may be computed. The thicknesses for the respective layers may then be determined as indicated below.

Solve for the SN required above the subbase material by using the resilient modulus of the subbase material (rather than the effective roadbed soil resilient modulus). Values of E_{SB} equal to 13,600 psi (for Rt. 2) and 12,100 psi (for Rt. 146), first stage reliability equal to 95 percent, w_{18} equal to 6.5×10^6 (for Rt. 2) and 17.0×10^6 (for Rt. 146), and ΔPSI_{TR} equal to 1.68 for Rt. 2 and 1.66 for Rt. 146 result in an $SN_1 = 4.0$ (for Rt. 2) and $SN_1 = 4.5$ (for Rt. 146). Thus, the asphalt concrete thickness required is:

Rt. 2

$$SN_1 = a_1D_1 \Rightarrow D_1 = SN_1 / a_1 = 3.9 / 0.45 = 8.7 \text{ (or } D_1^* = 9.0 \text{ inches)}$$

Rt. 146

$$SN_1 = a_1D_1 \Rightarrow D_1 = SN_1 / a_1 = 4.6 / 0.43 = 10.7 \text{ (or } D_1^* = 11.0 \text{ inches)}$$

Finally the thickness of subbase required is:

Rt. 2

$$D_3 = (SN_3 - SN_1^*) / (a_3m_3)$$

$$= (4.4 - 4.05) / (0.1*0.9) = 3.8$$

However, use $D_3^* = 12$ inches to meet the RI minimum requirement.

Rt. 146

$$D_3 = (SN_3 - SN_1^*) / (a_3m_3)$$

$$= (5.1 - 4.73) / (0.09*0.90) = 4.6$$

However, use $D_3^* = 12$ inches to meet the RI minimum requirement.

It should be recognized that, for flexible pavements, the structure is a layered system and was designed accordingly. The structure was designed in accordance to 1993 AASHTO Design Guide. First, the structural number required over the roadbed soil was computed. In the same way, the structural number required over the subbase layer and the base layer was also computed, using the applicable strength values for each. By working with differences between the computed structural number required over each layer, the maximum allowable thickness of any given layer was computed. For example, the subbase material would be equal to the structural number required over the subbase subtracted from the structural number required over the roadbed soil. In a like manner, the structural numbers of the other layers was computed.

8.5 PAVEMENT DESIGN USING DARWin

8.5.1 Introduction

The term DARWin comes from Pavement Design, Analysis, and Rehabilitation for Windows ("DARWin" 1993). In simple terms, DARWin is a computerized version of the pavement design models presented in AASHTO's Guide of Design for Pavement Structures 1993. However, the program actually does much more. For example, in flexible pavement design, DARWin allows the calculation of layer thicknesses by three user-selected methods, including an optimization scheme. The module on life-cycle costs is completely redesigned so that the user can input costs, in the same format as they are made available, for initial construction, rehabilitation, and maintenance. Outputs are customizable and can be presented six different ways. DARWin also has enhanced ESAL calculation procedures, report generation capabilities, display graphics, extensive on-line help, and many other exciting new features.

The pavement rehabilitation module, which is added to the release 2.01, incorporates the revised approach to pavement overlay design. The overlay module provides a fully automated means of performing all of the different overlay design calculations, including two FWD file reading and backcalculation.

DARWin permits the simultaneous running of multiple modules. Suppose you want to compare a number of different flexible pavement designs, each incorporating various combinations of different inputs. Opening a new module will allow you to do just that, and you can move from module to module, either with a click of the mouse or by using the windows option on the main option menu. Hence, DARWin is a very user-friendly and also helps in designing the pavement design structure without any cumbersome procedures.

The following are the outputs for the pavement design using the DARWin software for both Rt. 2 and Rt. 146.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations based on the findings and observations of this study are summarized below.

9.1 Conclusions

Resilient Modulus Testing

(1) Eight subgrade soils were selected from the field in Rhode Island, and were tested to determine the resilient modulus (M_R) in accordance with the procedure of AASHTO T292-91. Since specimens were not able to compacted or to be tested at some high field moisture contents, it was decided to develop regression equations to estimate M_R at any moisture contents.

(2) A series of M_R tests was performed at different moisture contents, temperatures and dry densities for two typical soils, which had field moisture and temperature data available. At normal and thawed conditions, the resilient modulus increased as the bulk stress increased with the relationship of $M_R = K_1\theta^{K2}$. However, this relationship was not clearly apparent at frozen condition in which the resilient modulus varied more depending upon the moisture content and deviator stress. It was also observed that the resilient modulus decreased as moisture content increased at a constant temperature, and it increased with the temperature decreasing at the certain moisture content as reported in the previous research report.

(3) To predict the resilient modulus under various environmental conditions, a series of multi linear regression analyses were conducted using the SAS computer program. Nine regression equations with four basic independent variables (bulk stress, moisture content, temperature and dry density) were developed for different test conditions, e.g., normal,

near-frozen, and normal plus near-frozen, based on the individual data set (Rt. 2 and Rt. 146) as well as combined data (Rt. 2 and Rt. 146).

(4) The monthly resilient moduli were determined with the average monthly temperature and moisture content at the average depth of significant stresses (ADSSs) for each site. The effective resilient moduli for each study site were determined using the combined M_R prediction equation for the normal condition and the AASHTO Guide procedure. The average effective resilient modulus for subgrade soils studied was 8.4 ksi with a standard deviation of 0.7 ksi.

<u>Cumulative 18 – kip ESAL</u>

(1) A procedure to estimate the cumulative 18-kip ESAL was developed utilizing the weight-in-motion (WIM) data in Rhode Island.

(2) The design lane traffic estimate for Rt. 2 and Rt. 146 were estimated as 10,549,906 and 28,749,901 18–kip ESALs for 20 years analysis period, respectively.

Layer Coefficient

(1) The layer coefficients of the subbase materials and asphalt concrete were determined using the direct method recommended by AASHTO. The layer coefficients for subbase materials ranged from 0.09 to 0.22 with an average of 0.15. Similarly the coefficient for bound layer ranged from 0.34 to 0.47 and the average is 0.39.

(2) The average layer coefficient for subbase materials, 0.15, determined from the present study was higher than the one (0.10) currently used in Rhode Island. Meanwhile the estimated average coefficient for bound layer, 0.39 was lower than the one (0.44) currently used in Rhode Island.

Drainage Coefficient

(1) The drainage coefficient of the subbase materials were determined using the method recommended by AASHTO. The drainage coefficients of the subbase layer ranged from 0.8 to 1.0 and the average is 0.9.

Application of Estimated Design Parameters

Since RIDOT uses the deep strength structure, the layer coefficients of bound layers in the nomograph procedure were weighted into one value: 0.45 and 0.43 for pavement structures of Rt. 2 and Rt. 146, respectively.

The application of estimated parameter to the AASHTO Guide resulted 9.0 in. bound and 12.0 in. granular subbase layers for Rt. 2 structure. Similarly the layer thickness of Rt. 146 structure were determined as 11 in. and 12 in. for bound and subbase layers. The higher thickness compared to the existing structure were mainly due to low drainage coefficients.

The application of estimated parameters to DARWin™ 2.01 software resulted 9.0 in. bound and 12.0 in granular subbase layers for Rt. 2 structure. Similarly the layer thickness of Rt. 146 structure were determined 11.0 in and 12.0 in. for bound and granular subbase layers, respectively

9.2 Recommendations

(1) Laboratory specimen preparation method should be improved to simulate the field condition closely. To avoid losing specimen strength and to simulate compaction effort better, the AASHTO TP46-94 procedure with the 6-in. diameter specimen can be considered for testing coarse-grained soils and subbase materials.

(2) The experimental study to develop prediction equation should be extended with more soils to determine if there are any other variations on their resilient moduli. More tests should be performed at frozen and thawed conditions to develop more accurate prediction equations for soil resilient moduli undergoing a freeze-thaw cycle. Further statistical analysis is also required.

(3) The average effective resilient modulus determined from this study can used as a tentative design value for subgrade soils which have similar characteristics and field conditions to those studied.

(4) The moisture-temperature cells embedded under pavement structures of Rt. 2 and Rt. 146 should be recalibrated or replaced, because some of the cells are not providing reasonable results and/or are dead.

(5) The procedure developed in the present study can be used to determine the cumulative ESALs for the design of pavement structures.

(6) In order to account for frost – heave in the serviceability loss, the values were assumed. More research work are required to determine the frost penetration and frost-heave effects.

(7) The procedure developed in the present study can be used to determine the drainage coefficients for the design of pavement structures. However, more research works are required to refine the procedure and coefficient values.

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Appendices are not published herein. However, copies of the appendices would be available from the author on request.